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# Extending geotechnical classification using excavation stability data in fine-grained alluvial deposits

**Emad Namavar** [1]✉

1. Instructor, Department of civil engineering, Faculty of engineering, Islamic Azad University of Chaloos, Chaloos, Iran. E-mail: Namavaremad@gmail.com

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**ABSTRACT**

Accurate geotechnical classification is essential for designing excavations in urban environments, where soil behavior is greatly affected by excavation-induced stresses. This study improves the geotechnical characterization of fine-grained alluvial deposits belonging to the youngest sedimentary unit (Unit D) in Rieben's classification system. A comprehensive investigation was conducted through borehole drilling, Standard Penetration Tests (SPT), pressuremeter testing, and laboratory experiments including triaxial, uniaxial, and direct shear tests. Excavation stability was evaluated using the Morgenstern–Price method under both short-term and long-term conditions. Based on the geotechnical parameters and slope stability simulations, Unit D was subdivided into three distinct zones (D1, D2, and D3) with different excavation behaviors. Zone D1, characterized by lower sand content, allows deeper vertical cuts, whereas the presence of sandy lenses in Zone D3 restricts excavation depth and requires gentler slopes. The findings provide an updated geotechnical classification framework for fine-grained alluvia, offering practical guidelines for safe excavation design and contributing to the broader understanding of alluvial systems in urban geotechnical engineering.

## Introduction

The rapid expansion of infrastructure and civil construction projects in Tehran highlights the need for robust geological classifications in engineering practice. Rieben (1966) proposed the first comprehensive classification of Tehran's alluviums dividing the coarse-grained deposits into four categories: A, B, C, and D. A represents the oldest deposits, and D represents the youngest. Table 1 summarizes the characteristics of these alluvial units using conventional terminology. Subsequent studies on these sediments include those by Gansser et al. (1962), Knill and Jones (1968), Vita-Finzi (1968, 1969), and Bassir (1971). Most of these works confirmed Rieben's classification without making significant modifications (Berberian et al., 1985). Later investigations, such as those by Cheshomi et al. (2008, 2009), evaluated the applicability of this classification in engineering projects, while other studies (Jafari et al., 1996; Asghari, 2003; Amini et al., 2014; Tempa et al., 2021; Moazami et al., 2024; Brilli et al., 2024; Khosronezhad et al., 2024; Akbarimehr et al., 2024) primarily addressed geological aspects of Tehran's alluviums. Incorporating geotechnical parameters into Rieben's classification enhanced its practical relevance for civil engineering applications. For example, Cheshomi et al. (2009) and Fakher et al. (2007) emphasized the practical integration of geological and geotechnical data.

Many researchers have investigated the engineering, geological and geotechnical

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properties related to large-scale civil and infrastructure projects such as dams, highways, and, in particular, tunnels. Delezalova (2002), for instance, examined numerical modeling approaches to ground movement caused by shallow tunneling. Similarly, Ozsan et al. (2006) carried out geological and geotechnical investigations along the Ankara Metro in Turkey, while Leca and New (2007) studied settlement behavior induced by tunneling in soft ground using both conventional and numerical techniques. In Iran, Alizadeh et al. (2021) explored the geotechnical aspects of subway stations in Tabriz Metro Line 2, and Azali et al. (2013) investigated mechanized tunneling using TBM in soft ground along the East–West section of Tehran Metro Line 7. Further studies have highlighted the role of local geotechnical properties in excavation and tunneling projects. Baghban et al. (2014) evaluated the influence of recent alluvial deposits on ground settlement during tunneling in Tehran. Mitew-Czajewska (2015) analyzed geological conditions, geotechnical parameters, and static performance of deep excavation retaining walls during Warsaw Metro construction. In another example, Sadjadi and Khalkhali (2018) assessed the geological and geotechnical challenges associated with Tehran Metro Line 7. Azarafza et al. (2018) investigated stochastic fracture network modeling as a key factor influencing geotechnical properties in the Kani-Sib tunnel (northwest Iran). Beyond Iran, Cueto et al. (2018) studied karst detection, prevention, and mitigation in Line 3 of the Riyadh Metro (Saudi Arabia), while Cueto et al. (2020) provided an engineering geological assessment of the same project based on geochemical, mineralogical, and petrographic analyses.

The present study has three main objectives: First, it aim to analyze alluvial sediments with respect to excavation stability using data obtained from borehole drilling, experimental testing, and numerical simulations. Second, it seeksto integrate these results with existing geological classifications. Finally, it intends to develop a framework for designing and constructing safe and stable excavations.

## Geographical and geological setting of Tehran's alluviums

Tehran, the capital city of Iran, is situated on a broad alluvial plain at the southern edge of the central Alborz Mountains. The regional topography is defined by north–south slopes and east–west variations in elevation. The landscape encompasses the Alborz Mountain front, mountain slopes, and the Tehran Plain from north to south.

The Alborz Range is primarily composed of tuff, limestone, and dolomite, and is characterized by steep relief. The area experiences heavy seasonal rainfall and is seismically active, resulting in complex and heterogeneous sedimentary layers.

From a geomorphological perspective, the Tehran Plain is subdivided into five units: mountains, foothills, old alluvial fans, young alluvial fans, and plains (Fakher et al., 2007). These units are composed of relatively recent alluvial deposits transported by rivers originating in the Alborz Mountains (Ghorbani and Ghorbani, 2021). Rieben (1966) classified these deposits into four categories (A–D), where units A and B are dominant in the mountainous and foothill zones. Unit C corresponds to older alluvial fans.Unit D represents young alluvial fans and plain deposits.

This study focuses specifically on the fine-grained deposits of unit D in the southwestern part of the plain. These sediments, derived from runoff channels, rivers, and recent alluvial cones, predominantly consist of clay and silt. Their spatial variability has significant implications for geotechnical behavior and excavation stability. Figure 1 illustrates the regional geological setting and the subdivision of the area based on slope stability analysis.

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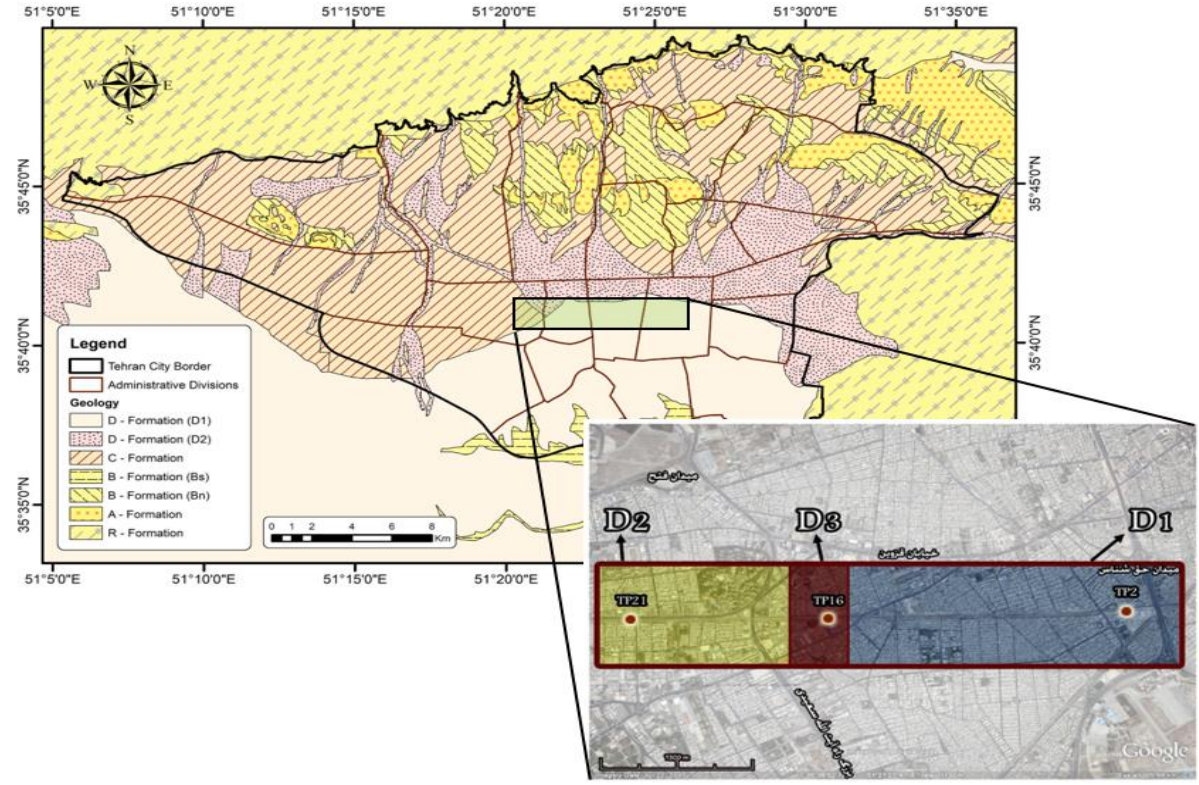



Fig. 1. The study area on the geological map of Tehran (up) and its zonation based on the excavation slope stability (down)

**Experimental and in situ tests**

A series of comprehensive laboratory and in situ tests was conducted to determine the geotechnical properties of the studied soils. Table 1 summarizes the types and standards of the performed tests, includingtriaxial, uniaxial, and direct shear tests, as well as grain size distribution, Standard Penetration Tests (SPT), in situ density measurements, and pressuremeter tests.

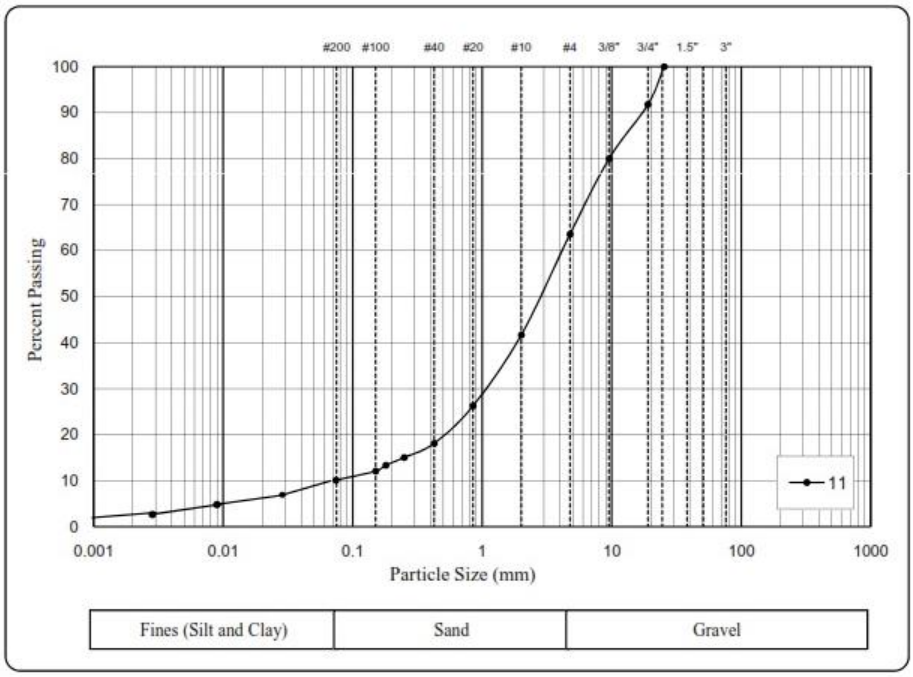
Table 1. List of conducted tests in this study

| Test name | Test type | | Number of tests | Standard |
|---|---|---|---|---|
| | In-situ | Laboratory | | |
| Triaxial test | - | ✓ | 35 | ASTMD4767-95 |
| Uniaxial test | - | ✓ | 11 | ASTM D7012 |
| Direct shear test | - | ✓ | 40 | ASTM D3080-90 |
| grain size distribution | - | ✓ | 132 | ASTM: D422-63 |
| SPT | ✓ | - | 18 | ASTM-D1586 |
| In situ density | ✓ | - | 148 | ASTM D1556 |
| Pressuremeter test | ✓ | - | 6 | ASTM D 4719 |

Grain size analysis revealed that the sediments consist of approximately 73.4% clay and silt, 23.6% sand, and 3% gravel-sized particles (Figure 2). The consolidated undrained triaxial tests, performed with pore pressure monitoring, provided key strength parameters such as effective cohesion (c′), internal friction angle (φ′), and undrained shear strength (Cu). An

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example result from a depth of 14 m is shown in Figure 3.




| No. | Depth | Gradation | | | | | | | | Att. Limits | | Classification |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | m | Gravel % | Sand % | Fines % | $D_{10}$ mm | $D_{30}$ mm | $D_{60}$ mm | $C_u$ | $C_c$ | LL % | PI % | USCS |
| 11 | 25.0-26.0 | 3.0 | 23.6 | 73.4 | | | | | | 36.8 | 16.4 | CL |

Fig. 2. One of the results obtained through grain size distribution test; the data is representative of the region's deposit collected from test pit TP16.

Figure 3 presents a representative result from a consolidated–undrained triaxial test performed at a depth of 14 meters in test pit TP-10. The effective strength parameters were determined from Mohr–Coulomb failure envelopes in conjunction with pore pressure measurements. For fine-grained soils under undrained conditions, φ′ is assumed to be zero for fine-grained soils, with Cu representing the total shear strength.

The outcomes of selected laboratory tests are summarized in Table 2. Direct shear tests were conducted according to ASTM D3080-90 on specimens with a height of 1 inch (25.4 mm). Each sample was placed in a shear box divided into two halves. A normal load was first applied to the specimen, followed by shear loading on the upper half of the box until failure occurred. Additional direct shear tests were carried out on saturated circular specimens with a diameter of 5 cm, using both slow and rapid shearing procedures. The corresponding results are reported in Table 2.

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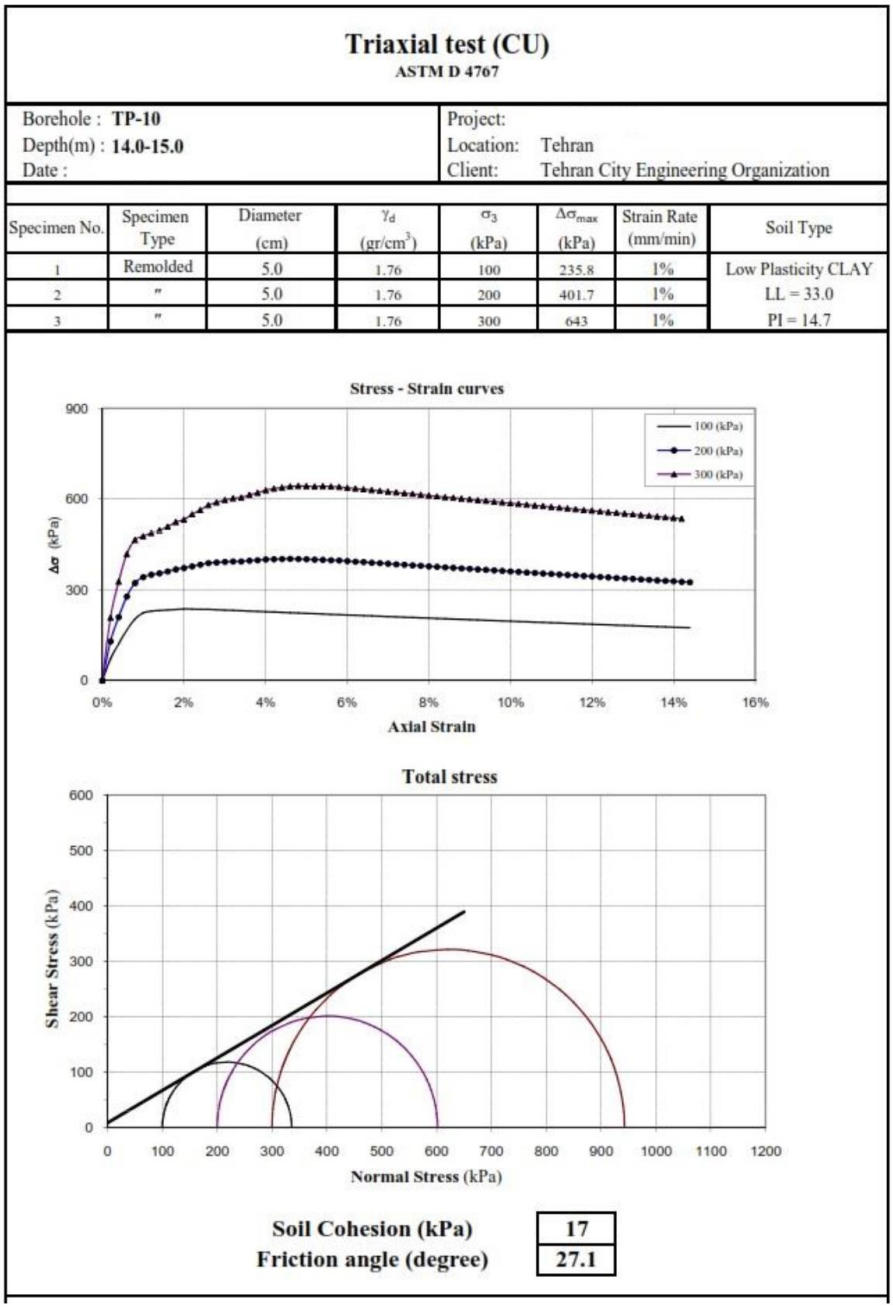



Fig. 3. A sample result of the consolidated-undrained triaxial test ($C_U$) performed in the study

A uniaxial compressive strength (UCS) test conducted on saturated clay samples can be considered equivalent to an undrained, unconsolidated triaxial test. In this case, the confining pressure is zero, and the axial load is applied incrementallyuntil failure occurs. At the failure point, the minimum principal stress ($\sigma_3$) equals zero, while the maximum principal stress corresponds to $\sigma_1$. The undrained shear strength ($C_u$) is then calculated as half of $\sigma_1$, in accordance with ASTM D7012.

The results of the in situ tests are summarized in Tables 3–5. A standard penetration test (SPT) was performed following ASTM D1586 (corrected in 1984). In this test, a standard sampler with an outer diameter of 2 inches and an inner diameter of 1 inch is driven into the soil by a 140-pound (63.5 kg) hammer dropped from

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a height of 30 inches (76.2 cm). Ideally, 60% of the hammer's energy is transmitted to the drill rods. The number of hammer blows required to penetrate the sampler is recorded over three successive 15-cm intervals, with the sum of the blows for the last two intervals reported as the SPT N-value.

The SPT N-value can be used to estimate various geotechnical properties, including the relative density of sands, bearing capacity, potential for dynamic settlement, susceptibility to liquefaction, and unconfined compressive strength of soils (in kPa). Since the study area is mostly covered by fine-grained deposits—mainly low-plasticity clay (CL), the correlation proposed by Terzaghi et al. (1996) between SPT N-values and undrained shear strength for such soils was employed to estimate the physical parameters of the subsurface soils.

Table 2. Results of some laboratory tests in this project

| Borehole No. | Depth | Test type | Undrained shear strength ($C_U$; kg/cm$^2$) | Friction angle (φ′;°) | Cohesion (C′; kg/cm$^2$) | Sample type |
|---|---|---|---|---|---|---|
| TP2 | 8 | Triaxial test (Cu) | - | 21 | 0.75 | Undisturbed |
| | 6.2 | Uniaxial test | 0.51 | - | - | Undisturbed |
| | 3 | Direct shear test | - | 32 | 0.27 | Undisturbed |
| TP5 | 13 | Triaxial test (Cu) | - | 31 | 0.03 | Disturbed |
| | 4.2 | Uniaxial test | 0.36 | - | - | Undisturbed |
| | 9 | Direct shear test | - | 25 | 0.48 | Undisturbed |
| TP10 | 10 | Triaxial test (Cu) | - | 28 | 0.17 | Undisturbed |
| | 12.2 | Uniaxial test | 0.56 | - | - | Undisturbed |
| | 5 | Direct shear test | - | 32 | 0.08 | Disturbed |
| TP15 | 8 | Triaxial test (Cu) | - | 28 | 0.30 | Undisturbed |
| | - | Uniaxial test | - | - | - | Undisturbed |
| | 11 | Direct shear test | - | 26 | 0.40 | Undisturbed |
| TP20 | 12 | Triaxial test (Cu) | 0.38 | - | - | Undisturbed |
| | - | Uniaxial test | - | - | - | Undisturbed |
| | 13 | Direct shear test | - | 31 | 0.02 | Disturbed |

Undrained shear strength was determined using the results of the SPT test (Terzaghi et al. 1996).

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Table 3. Results of in situ density tests in selected test pits

| Pit No. | Depth (m) | Dry density (gr/cm$^3$) | Moisture percentage (%) |
|---|---|---|---|
| TP2 | 4 | 1.48 | 17.70 |
| | 6 | 1.51 | 16.46 |
| | 8 | 1.50 | 18.89 |
| TP14 | 7 | 1.64 | 20.39 |
| | 9 | 1.73 | 17.99 |
| | 11 | 1.64 | 21.17 |
| TP19 | 3 | 1.74 | 14.41 |
| | 5 | 1.74 | 16.27 |
| | 7 | 1.63 | 16.88 |

Table 4. Results of SPT test in BH2

| Test No. | Depth (m) | $N_{SPT}$ | $N_{CORECTED}$ | Undrained shear strength ($C_U$; kg/cm$^2$) | Unconfined compressive strength ($U_C$; kg/cm$^2$) | Soil Type |
|---|---|---|---|---|---|---|
| 1 | 2 | 19 | - | - | - | GC-GM |
| 2 | 4 | 18 | 9 | 0.57 | 1.14 | CL |
| 3 | 6 | 12 | 6 | 0.37 | 0.74 | CL |
| 4 | 8 | 11 | 5-6 | 0.31- 0.37 | 0.62- 0.74 | CL |
| 5 | 10 | 10 | 5 | 0.31 | 0.62 | CL |
| 6 | 12 | 11 | 5-6 | 0.31- 0.37 | 0.62- 0.74 | CL |
| 7 | 14 | 8 | 4 | 0.25 | 0.50 | CL |
| 8 | 16 | 9 | 4-5 | 0.25- 0.31 | 0.50- 0.62 | CL |
| 9 | 18 | 10 | 5 | 0.31 | 0.62 | CL |

The pressuremeter test was another in situ investigation conducted in this study. During the test, either the pressure or the volume is maintained constant for a set period of time, usually 30–60 seconds. The soil's creep response is recorded by measuring deformations at different times during each pressure or volume increment. In this study, readings were taken at 30 and 60 seconds, following ASTM D4719. These results were then used to construct pressure–volume curves.

Pressuremeter curves provide several parameters that can be correlated with soil properties or used directly in geotechnical design. The undrained shear strength of the soil was estimated using the pressuremeter test and empirical formulations proposed by Briaud (2019).

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Table 5. Calculated undrained shear strength from pressuremeter tests in BH2 and BH3

| Borehole name | Test No. | Depth (m) | Limit pressure ($PL^*$; kg/cm$^2$) | Undrained shear strength ($C_U$; kg/cm$^2$) | Soil type |
|---|---|---|---|---|---|
| BH2 | 1 | 3.35 | 6.19 | 0.94 | CL |
| | 2 | 7 | 5.37 | 0.82 | CL |
| | 3 | 11 | 4.78 | 0.73 | CL-ML |
| BH3 | 4 | 4 | 7.29 | 1.12 | CL |
| | 5 | 9 | 8.99 | 1.38 | CL |
| | 6 | 14 | 8.94 | 1.37 | CL-ML with sand |

Notably,no groundwater table was observed in any of the drilled boreholes or test pits. The deepest borehole reached 30 meters, and the maximum thickness of the investigated formation (D) was approximately 20 m.

## Geological profile

The geological profile of the study area was determined using grain size distribution tests and the Unified Soil Classification System (USCS), up to a depth of 20 meters (Fig. 4). The results indicate that the predominant soil type is low-plasticity clay (CL), howeverdiscontinuous layers and lenses of silty clay, silt, and silty sand were also identified throughout the stratigraphy. This heterogeneous distribution implies that, while fine-grained soils dominate while occasional coarse-grained lenses are embedded within the clay matrix. Such heterogeneities can significantly influence excavation behavior and overall slope stability. Each soil layer was classified according to USCS criteria, which provides a basis for correlating geological characteristics with geotechnical performance.

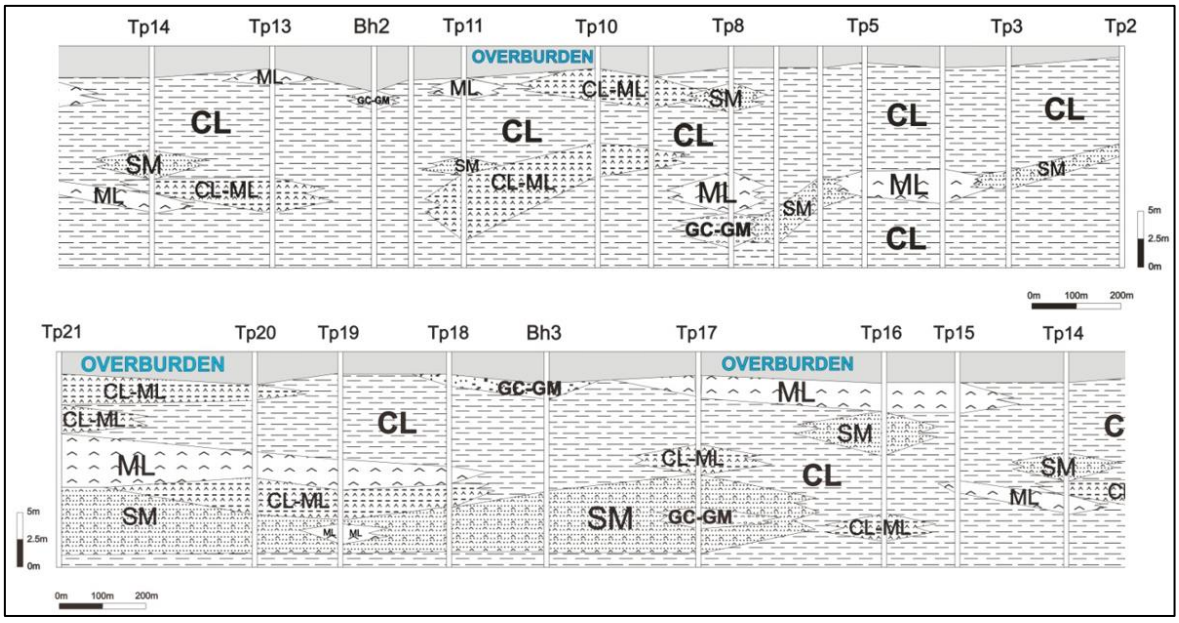



Fig. 4. Geological profile of the area; clay is the main constituent of the ground material. In this case, coarse-grained deposits are found as diffused lenses (layers named based on unified classification).

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**Geotechnical profile**

This section describes howthe development of the geotechnical profile was developed by integrating geological observations with the results of both laboratory and in situ tests. Instead of defining subsurface layers based solely on soil type, stratification was established based on the consistency of key engineering parameters, including moisture content, unit weight, cohesion, friction angle, and undrained shear strength.

Initially, distinct soil units were grouped according to uniform engineering characteristics, regardless of their geological classification. These preliminary groupings were then refined using undrained shear strength data derived from in situ testing and field observations. The final geotechnical profile was constructed by combiningall available information and applying sound engineering judgment. Due to the limited number of in situ tests, laboratory data were considered the primary source of information, and field test results were used as supplementary for validation.

Figure 5 illustrates the geotechnical zoning of the study area, comprising seven layers. Table 6 summarizes the engineering parameters for each layer. The stratigraphy includes various soil types, such as CL, CL-ML, SM, and ML-SM, each exhibiting distinct mechanical properties. This classification provides a more reliable basis for assessing subsurface conditions, which is essential for excavation and construction planning.

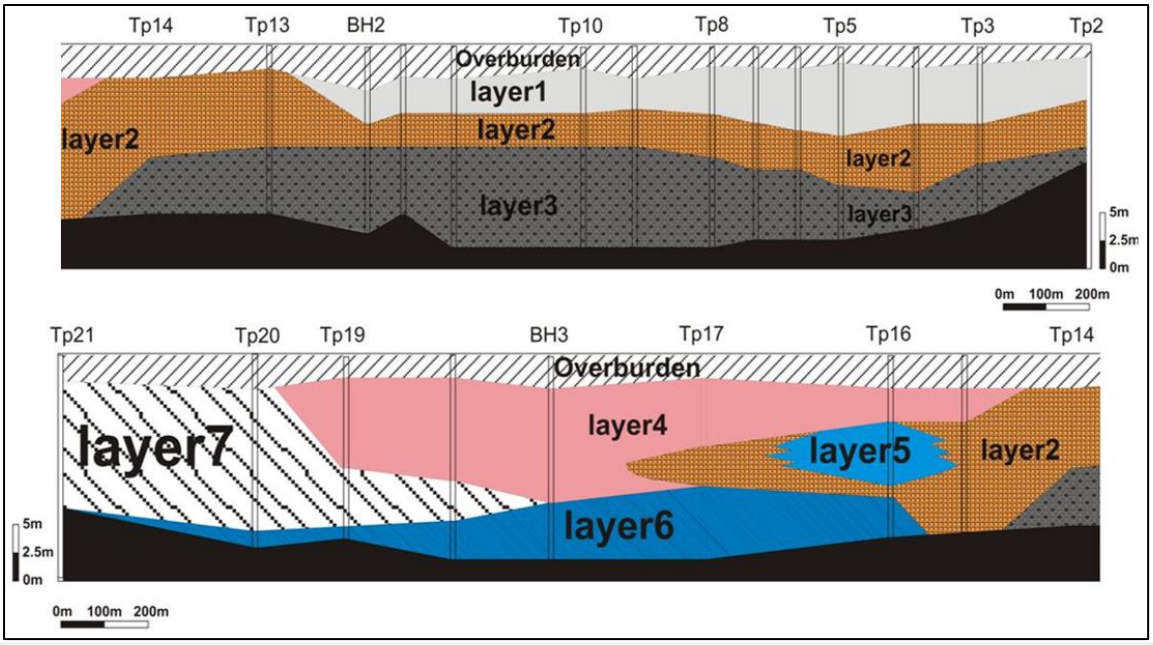



Fig. 5. Geotechnical profile of the study area

Table 6. Engineering parameters of the studied soil layers

| Layer No. | Soil type | Moisture percentage (%) | $\gamma_d$ (kN/m$^3$) | $\gamma_{sat}$ (kN/m$^3$) | C′ (kPa) | φ′ (°) | $C_u$ (kPa) |
|---|---|---|---|---|---|---|---|
| 1 | CL-ML | 16.9 | 16.2 | 18.9 | 24 | 29 | 29 |
| 2 | CL | 17.0 | 16.8 | 19.6 | 42 | 30 | 52 |
| 3 | CL | 19.5 | 17.4 | 20.8 | 38 | 24 | 46 |

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| 4 | CL-ML | 18.1 | 17.2 | 20.3 | 20 | 33 | 26 |
| 5 | SM | 20.4 | 16.9 | 20.4 | 14 | 25 | 19 |
| 6 | ML-SM | 12.7 | 19.1 | 21.5 | 17 | 32 | 21 |
| 7 | CL-ML | 14.8 | 18.0 | 20.6 | 27 | 23 | 33 |

**Slope stability analysis**

The Limit Equilibrium Method (LEM) is among the most widely used approaches for evaluating the stability of slopes and excavations. Despite its extensive application, the fundamental assumptions of LEM are often misinterpreted, which can sometimes lead to overestimated safety assessments. At its core, LEM relies on static equilibrium equations applied to a potential failure mass, which is typically subdivided into vertical slices.

Different limit equilibrium method (LEM) approaches vary in how they account for interslice forces and formulate the equilibrium equations. Among the most widely used methods are those proposed by Spencer (1967) and Morgenstern–Price (1965), both of which satisfy force and moment equilibrium while incorporating interslice forces. The Morgenstern–Price method is particularly favored due to its computational efficiency and flexibility.

Advancements in computer-based modeling have made it possible to perform rigorous limit equilibrium analyses using software such as GeoStudio. This study utilized the SLOPE/W module of GeoStudio. The analysis required parameters including the dry unit weight (for zones above the groundwater table), internal friction angle (under drained conditions), cohesion (for long-term stability), saturated unit weight (for zones below the groundwater table), and undrained shear strength (for short-term stability assessment).

**Long-term stability analysis of excavations**

Long-term stability analyses of excavations were performed using the dry unit weight, internal friction angle, and cohesion of soil samples as input parameters. In this study, the semi-sinusoidal function of the Morgenstern–Price method was applied. This approach accounts for all forces acting on each slice or segment, relying on both moment and force equilibrium. The potential failure mass was discretized into 50 slices for all analyses.

To provide a more realistic and accurate prediction of the critical slip surface, the failure surface optimizer option was activated in all analyses. The software evaluated 4,851 trial surfaces and selected the one with the lowest factor of safety (FS). A critical aspect of the optimization procedure is the method used to adjust the endpoints of the line segments. SLOPE/W moves these points within an elliptical search area using a statistical random walk procedure based on the Monte Carlo simulation method. The optimization outcome is somewhat dependent on the initial position of the trial slip surface, as the elliptical search area is defined relative to the starting slip surface. Consequently, an inadequate choice of the initial surface can restrict the final optimized slip surface.

Figure 6 illustrates a representative software output for the excavation at test pit TP5, to a depth of 12 m with a 60° slope comprising three geological layers. Long-term excavation stability was assessed for all 22 pits and boreholes of varying depths. Overall, the results demonstrate that excavation stability can be reliably predicted using the developed geotechnical profile.

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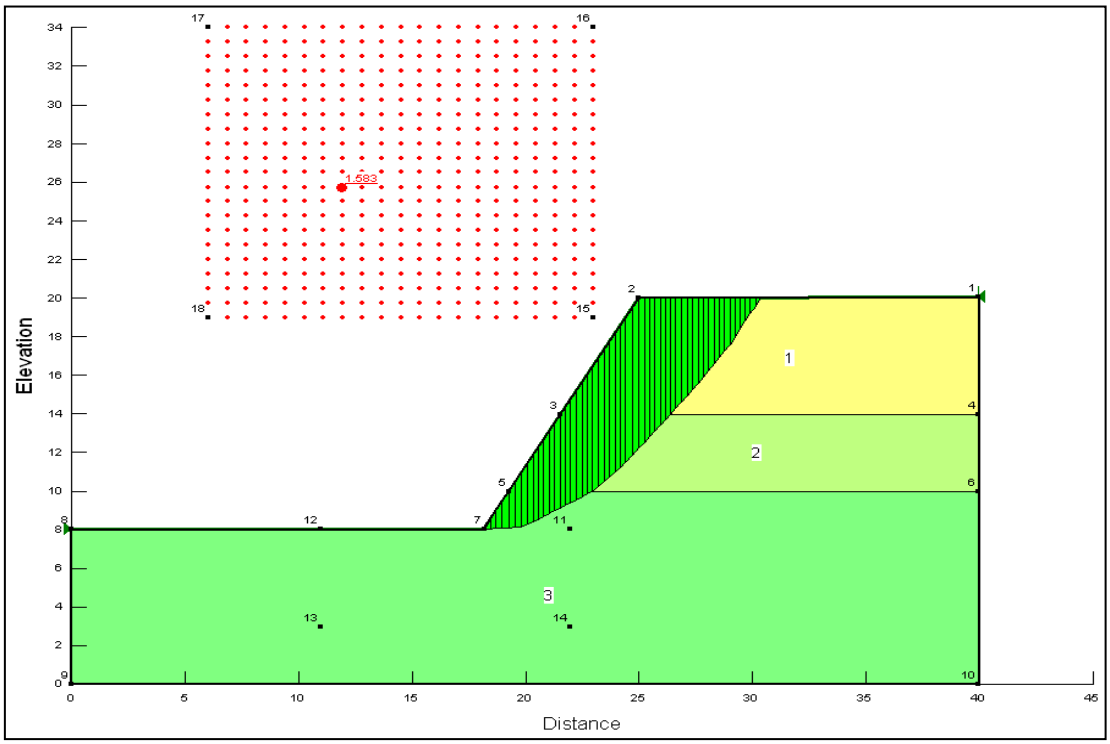



Fig. 6. A sample of software output for an excavation with a depth of 12 m and a face angle of 60° (units are in meters)

This step involved calculating the factor of safety (FS) for different slope in order to determine an appropriate profile for excavation stability.. Based on soil mechanics guidelines, an FS of 1.5 was adopted as the minimum threshold for safe and stable excavation. Analyses commenced with a slope angle of 90°, incrementally increasing the excavation depth by 1-m steps while determining the corresponding FS. When FS values fell below 1.5, the slope angle was reduced to achieve the required stability. Geotechnical profiles at different depths were evaluated by gradually decreasing the slope angle in 5° increments.

Analyses were conducted at depths of up to 12 m for each segment. Overburden was neglected in all cases. For situations where the factor of safety FS remained below 1.5, a trial-and-error approach was applied to identify the maximum slope angle that satisfied the stability criterion. The results for the three zones reveal distinct excavation stability patterns. Figure 7 presents the excavation stability graphs for these zones at different depths corresponding to FS = 1.5. According to the figure, a slope angle of 90° is sufficient to achieve FS = 1.5 up to a depth of 4 m for all zones. These areas are designated as D1 (zone 1), D2 (zone 2), and D3 (zone 3) in Figure 1.

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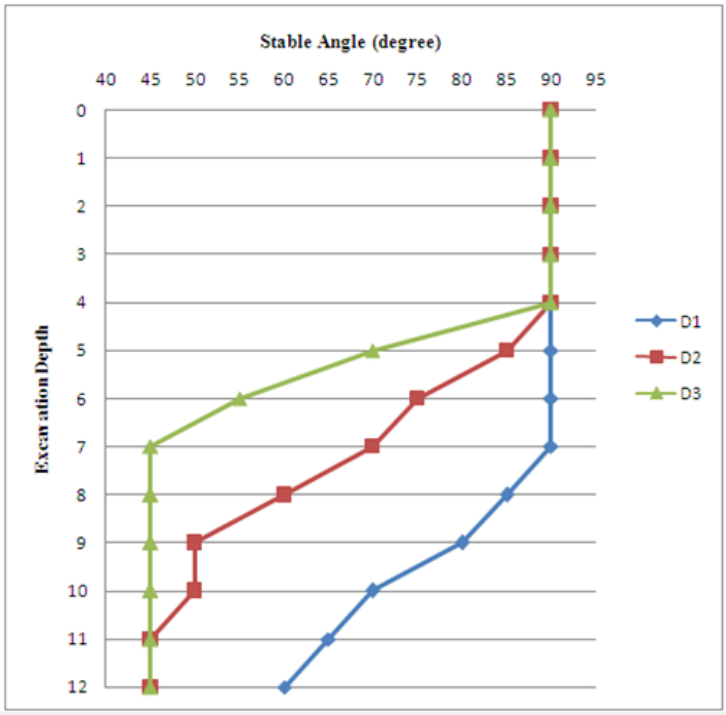



Fig. 7. The graphs representing the excavation patterns in zones $D_1$, $D_2$, and $D_3$

Figure 8 illustrates an excavation in zone D1 with a height of 6 m and a face slope of 80°, confirming the excavation pattern established for this zone. Back analyses were conducted on the excavated slopes to validate the geotechnical profiles and ensure consistency between laboratory and in situ test results and the predicted excavation patterns.

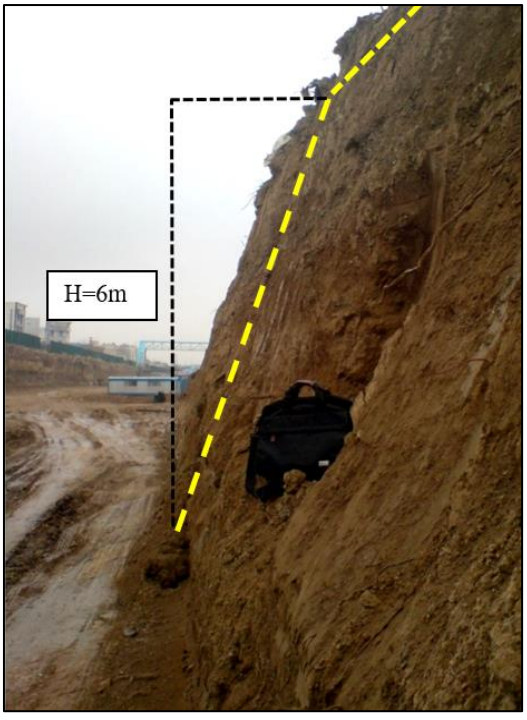



Fig. 8. The stable excavation wall with the approximate depth of 6 m and 80° in zone $D_1$

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**Short-term stability analysis of excavations**

This section presents analyses conducted under the most critical conditions. For example, it assumesthat the groundwater table coincides with the ground surface. With a constant excavation face angle, variations in the factor of safety (FS) were evaluated as a function of depth. Input parameters for this analysis included the soil's saturated unit weight and undrained shear strength, while the internal friction angle was set to zero to account for the most critical scenario. Similar to the long-term stability analyses, the semi-sinusoidal function of the Morgenstern–Price method was employed in conjunction with the failure surface optimizer.

Figure 9 presents the analytical results, showing FS values versus excavation depth. Careful examination of these graphs allows for the identification of geological zones (D1, D2, and D3) and provides the required FS for stable excavation within each zone. For instance, in zone D1, FS exceeds 1.5 at depths of approximately 4–5 m with a vertical face, whereas zones D2 and D3 reach the same FS at a depth of 3 m. It is noteworthy that no groundwater was observed in any of the test pits or boreholes. Consequently, short-term stability analyses serve primarily as a complementary dataset for geotechnical classification in excavation studies.

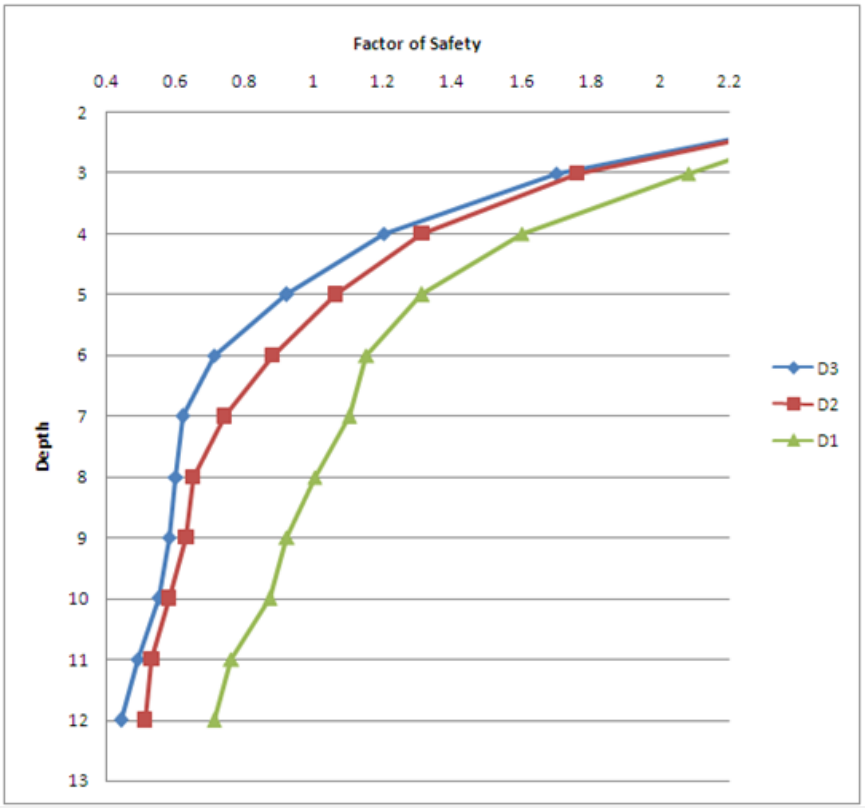



Fig. 9. Factor of safety as a function of depth with vertical slope face for the short-term stability analysis at zones $D_1$, $D_2$ and $D_3$

**Supplementing engineering information to previous geotechnical classifications**

Fakher et al. (2007) incorporated engineering parameters of alluvial deposits as supplementary elements in geological classifications. Their study primarily focused on the coarse-grained portions of the Tehran alluvium (members A and C) and, to a lesser extent, on members B and D. The present study complements these findings by providing additional engineering parameters for member D of the Tehran alluvium and subdividing this member based on excavation stability.

These complementary data were integrated with the existing geological classifications. As

summarized in Table 7, member D of the Tehran alluvium can be divided into three distinct zones from a slope stability perspective. Moreover, excavation stability patterns were established for each of the three zones of member D, with one column indicating the stable excavation depth for a vertical face and another specifying the stable face slope for a 10 m excavation in each zone.

Table 7. Appending engineering parameters and excavation stability pattern in addition to the previous classifications of alluvium D in south Tehran

| | Geological features | | | | Approximate values of engineering properties | | Excavation Pattern (Long-Term Analysis) | | |
|---|---|---|---|---|---|---|---|---|---|
| | Grain shape | Contact shape | Cement | Grain size | C (kPa) | φ(°) | Region | Stable depth for vertical wall (m) | Stable dip for 10-meter excavation (°) |
| Dalluviums | Round | Floating | Uncemented | CL-ML | 25-45 | 23-33 | $D_1$ | 7-8 | 70-80 |
| | | | | | | | $D_2$ | 4-5 | 50-55 |
| | | | | SM | 10-18 | 23-27 | $D_3$ | 4 | 45 |

## Conclusion

Enhancing geological classifications with engineering data provides valuable insights for safely designing and constructing civil infrastructure. This study focused on the fine-grained D member of the Tehran alluvium and developed a framework for assessing excavation stability based on laboratory and in situ testing.Three distinct zones (D1, D2, and D3) were identified based on excavation behavior. Zone D3 exhibited the lowest stability due to the presence of sandy layers at depths of 3–7 m, which reduced the factor of safety (FS) and required gentler slope angles. Conversely, zone D1 permitted steeper slopes and deeper vertical cuts. Short-term analyses under critical conditions confirmed the validity of this zone-based classification.

Key soil parameters, including undrained shear strength, cohesion, internal friction angle, and unit weight, were obtained from uniaxial compressive strength (UCS) tests, standard penetration tests (SPT), and pressuremeter tests. Integrating these geotechnical data with geological observations allowed for an accurate profile of the subsurface and identification of heterogeneities, both of which are critical for predicting Limit equilibrium analyses using the Morgenstern–Price method offered both long-term and short-term stability evaluations. Excavation stability patterns for the identified zones highlighted maximum allowable slope angles and depths to maintain FS $\geq$ 1.5. Back analyses on existing excavations confirmed the reliability of the predicted stability patterns.

Overall, this study provides a systematic approach to geotechnical profiling and excavation stability assessment. This approach provides practical guidelines for desighning safe excavation in the Tehran alluvium and can be applied to other fine-grained depositional environments.

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# توسعه رده بندی های ژئوتکنیکی با استفاده از داده های پایداری شیب در رسوبات دانه ریز

عماد نام آور[۱] ✉

۱. گروه مهندسی عمران، دانشگاه آزاد اسلامی واحد چالوس، چالوس، ایران. رایانامه: Namavaremad@gmail.com

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**کلیدواژه‌ها:**
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**چکیده**

طبقه‌بندی ژئوتکنیکی دقیق، برای طراحی گودبرداری در محیط‌های شهری ضروری است، زیرا رفتار خاک به‌شدت تحت تأثیر تنش‌های ناشی از گودبرداری قرار دارد. این پژوهش به بازنگری در ویژگی‌های ژئوتکنیکی رسوبات آبرفتی ریزدانه مربوط به جوان‌ترین واحد رسوبی (واحد D) در طبقه‌بندی ریبن پرداخته است. برنامه جامع مطالعاتی شامل حفاری گمانه‌ها، آزمایش نفوذ مخروط (SPT)، آزمایش پرسیومتری و آزمایش‌های آزمایشگاهی سه‌محوری، تک‌محوری و برش مستقیم انجام گرفت. پایداری گودبرداری با استفاده از روش Morgenstern–Price در شرایط کوتاه‌مدت و بلندمدت ارزیابی شد. بر اساس پارامترهای ژئوتکنیکی و پایداری شیب، واحد D به سه ناحیه متمایز (D1، D2 و D3) با رفتارهای متفاوت در گودبرداری تقسیم گردید. ناحیه D1 که دارای مقدار ماسه کمتر است، امکان ایجاد ترانشه های قائم عمیق‌تر را فراهم می‌کند، در حالی که وجود عدسی‌های ماسه‌ای در ناحیه D3 عمق گودبرداری را محدود کرده و نیازمند شیب‌های ملایم‌تر می‌باشد. نتایج این مطالعه یک چارچوب به‌روز برای طبقه‌بندی ژئوتکنیکی آبرفت‌های ریزدانه ارائه می‌دهد که علاوه بر ارائه دستورالعمل‌های عملی برای طراحی ایمن گودبرداری، به درک جامع‌تر از سامانه‌های آبرفتی در مهندسی ژئوتکنیک شهری کمک می‌کند.

## مقدمه

پروژه‌های مهندسی عمران شهری در رسوبات آبرفتی اغلب با چالش‌های ژئوتکنیکی قابل توجهی مواجه می‌شوند که ناشی از شرایط زیرسطحی پیچیده و ناهمگن است. آبرفت‌های تهران، که از رسوبات دانه‌ریز و دانه‌درشت تشکیل شده‌اند، مجموعه‌ای از عدم قطعیت‌های مهندسی را ایجاد می‌کنند که برای اجرای ایمن حفاری و ساخت‌وساز باید به آنها توجه شود. در حالی که مطالعات پیشین عمدتاً بر اعضای دانه‌درشت آبرفت‌های تهران متمرکز بوده‌اند، رسوبات دانه‌ریز، به‌ویژه عضو D ، توجه کمتری دریافت کرده‌اند، اگرچه تأثیر قابل‌توجهی بر پایداری شیب‌ها و رفتار حفاری دارند (Fakher et al., 2007; Cheshmi et al., 2008). ماهیت ناهمگن این رسوبات شامل خاک‌های رس با پلاستیسیته کم، رس‌های سیلتی، سیلت و عدسی‌های ماسه‌ای پراکنده است که لایه‌هایی با خواص مکانیکی متفاوت ایجاد می‌کنند. این تفاوت‌ها پارامترهایی همچون مقاومت برشی بدون زهکشی، چسبندگی و سختی را تحت تأثیر قرار می‌دهند و در نهایت بر پایداری حفاری، طراحی شیب و ایمنی کلی ساخت‌وساز تأثیرگذار هستند. بنابراین، شناسایی دقیق ژئوتکنیکی نیازمند ادغام نتایج آزمایشگاهی، مشاهدات میدانی و مطالعات زمین‌شناسی است. در سال‌های اخیر، پیشرفت‌های حاصل در مدل‌سازی عددی و روش‌های تعادل حدی امکان ارزیابی قابل اعتمادتر پایداری شیب‌ها را

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تحت شرایط کوتاه‌مدت و بلندمدت فراهم کرده است (Spencer, 1967; Morgenstern and Price, 1965; Sun et al., 2023). ترکیب اندازه‌گیری‌های میدانی، آزمایش‌های آزمایشگاهی و تحلیل‌های عددی، به مهندسان امکان می‌دهد تا ناهمگنی‌ها و ویژگی‌های خاص سایت را لحاظ کنند و راهنمایی‌های دقیق‌تری برای طراحی حفاری ارائه دهند. مطالعه حاضر با تمرکز بر عضو دانه‌ریز D آبرفت‌های تهران، تلاش می‌کند تا چارچوب عملی برای پایداری حفاری ارائه دهد. با ادغام داده‌های آزمایشگاهی (آزمایش مقاومت فشاری تک‌محوره و فشارسنج)، اندازه‌گیری‌های میدانی (SPT و فشارسنج) و تحلیل‌های عددی پایداری شیب (SLOPE/W با روش Morgenstern–Price)، سه ناحیه متمایز D1، D2 و D3 با رفتارهای حفاری متفاوت شناسایی شدند. این رویکرد یک طبقه‌بندی مبتنی بر مهندسی ارائه می‌دهد که فراتر از طبقه‌بندی زمین‌شناسی سنتی است و راهنمایی عملی برای طراحی شیب‌ها، تعیین عمق‌های حفاری ایمن و ارزیابی ضریب ایمنی فراهم می‌کند. در مجموع، این مطالعه به درک بهتر رفتار رسوبات دانه‌ریز آبرفتی تهران کمک می‌کند و اهمیت ترکیب داده‌های زمین‌شناسی، آزمایشگاهی و عددی را برای برنامه‌ریزی ایمن و قابل اعتماد حفاری در محیط‌های شهری نشان می‌دهد.

## مواد و روش ها

این مطالعه از ترکیبی از تحقیقات آزمایشگاهی و میدانی برای استخراج پارامترهای حیاتی خاک استفاده کرد. آزمایش‌های مقاومت فشاری تک‌محوره (UCS) بر نمونه‌های رس اشباع‌شده به‌عنوان آزمایش‌های سه‌محوری بدون زهکشی و بدون تحکیم تفسیر شدند، که امکان تعیین مقاومت برشی بدون زهکشی را فراهم می‌کند. آزمایش‌های نفوذ استاندارد (SPT) مطابق با ASTM D1586 انجام شد و داده‌های تجربی در مورد مقاومت خاک را ارائه داد، که امکان برآورد پارامترهای کلیدی مانند چگالی نسبی، ظرفیت باربری، پتانسیل روانگرایی و مقاومت برشی بدون زهکشی را فراهم ساخت. آزمایش‌های فشارسنج (Pressuremeter) مطابق با ASTM D4719 داده‌های تکمیلی در مورد سختی درجا و رفتار تنش-کرنش لایه‌های خاک، شامل اندازه‌گیری کرنش خزشی در سطوح فشار افزایشی، ارائه داد. پروفایل زمین‌شناسی تا عمق ۲۰ متر توسعه یافت و بر اساس آزمون‌های دانه‌بندی و سیستم طبقه‌بندی یکپارچه خاک (USCS)شکل گرفت. پروفایل، رس با پلاستیسیته کم (CL)را به‌عنوان نوع غالب خاک شناسایی کرد، که با عدسی‌های پراکنده از رس سیلتی، سیلت و ماسه سیلتی همراه بود. پروفایل‌های ژئوتکنیکی با ادغام پارامترهای مهندسی مانند میزان رطوبت، وزن واحد، چسبندگی، زاویه اصطکاک داخلی و مقاومت برشی بدون زهکشی—در چارچوب زمین‌شناسی توسعه یافتند، که منجر به یک طبقه‌بندی پیش‌بینی‌کننده‌تر شد و فراتر از تشخیص بصری نوع خاک عمل کرد. تحلیل‌های پایداری شیب با استفاده از روش تعادل حدی (LEM) و رویکرد Morgenstern–Price که در ماژول SLOPE/W نرم‌افزار GeoStudio پیاده‌سازی شده بود، انجام شد. شرایط بلندمدت و کوتاه‌مدت هر دو مورد بررسی قرار گرفتند: پایداری بلندمدت با استفاده از وزن واحد خشک، زاویه اصطکاک داخلی و چسبندگی تحلیل شد، در حالی که پایداری کوتاه‌مدت، وزن واحد اشباع و مقاومت برشی بدون زهکشی را تحت شرایط بحرانی (با فرض صفر بودن زاویه اصطکاک داخلی) لحاظ کرد. بهینه‌سازی سطح گسیختگی با استفاده از روش تصادفی مبتنی بر مونت‌کارلو در محدوده جستجوی بیضوی انجام شد و تحلیل‌ها در زوایای شیب و عمق‌های حفاری مختلف برای تعیین ضریب ایمنی (FS) صورت گرفت. تحلیل‌های

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بازگشتی بر روی ترانشه‌های حفاری‌شده نیز الگوهای پایداری پیش‌بینی‌شده را تأیید کردند.

## بحث و نتایج

تحقیقات ژئوتکنیکی نشان داد که توزیع خاک‌ها در عضو D آبرفت‌های تهران ناهمگن است. خاک رس با پلاستیسیته کم (CL) غالب است و با عدسی‌های پراکنده از رس سیلتی، سیلت و ماسه سیلتی همراه می‌باشد. این ناهمگنی‌ها به‌طور قابل توجهی بر رفتار حفاری تأثیر می‌گذارند، همان‌گونه که نتایج آزمایش‌های آزمایشگاهی و میدانی نشان می‌دهند. آزمایش‌های مقاومت فشاری تک‌محوره (UCS) و فشارسنج نشان‌دهنده تغییرات مقاومت برشی بدون زهکشی و سختی در لایه‌ها بود و ضرورت تحلیل مبتنی بر ناحیه (Zone specific) را به جای اعمال یک مجموعه پارامتر یکنواخت در کل سایت برجسته می‌کند. سه ناحیه متمایز D1، D2 و D3 بر اساس معیارهای پایداری حفاری شناسایی شدند. ناحیه D1 بیشترین مقاومت برشی بدون زهکشی و چسبندگی را نشان داد و اجازه می‌داد شیب‌های تند و ترانشه‌های عمودی تا عمق ۴–۵ متر با حفظ ضریب ایمنی ≥ (FS)۱٫۵ اجرا شوند. تحلیل پایداری بلندمدت نشان داد که ترکیب وزن واحد خشک، زاویه اصطکاک داخلی و چسبندگی برای حفظ پایداری در شرایط معمول حفاری کافی است. تحلیل کوتاه‌مدت، با در نظر گرفتن وزن واحد اشباع و مقاومت برشی بدون زهکشی، این نتایج را تحت شرایط بحرانی تأیید کرد و FS حتی در سناریوهای محافظه‌کارانه بالاتر از حد ایمنی باقی ماند. ناحیه D2 دارای پایداری متوسط بود. در حالی که ترانشه‌های عمودی تا حدود ۳ متر قابل نگهداری بودند، حفاری‌های عمیق‌تر نیاز به کاهش تدریجی زاویه شیب به میزان ۵–۱۰ درجه برای دستیابی به FS ≥ ۱٫۵ داشتند. داده‌های فشارسنج و همبستگی‌های حاصل از SPT لایه‌های ضعیف موضعی را نشان دادند که باعث کاهش مقاومت در برابر لغزش بالقوه می‌شدند. این نتایج اهمیت شناسایی عدسی‌های نازک و با مقاومت کم در رسوبات عمدتاً چسبنده را نشان می‌دهد، زیرا حتی لایه‌های کوچک می‌توانند پایداری کلی شیب و طراحی حفاری را تحت تأثیر قرار دهند. ناحیه D3 کمترین پایداری را داشت، به دلیل وجود لایه‌های ماسه‌ای در عمق ۳–۷ متر. این لایه‌ها چسبندگی و مقاومت اصطکاکی کمتری نشان دادند و منجر به کاهش قابل توجه FS برای ترانشه‌های عمودی شدند. تحلیل‌ها نشان داد که حداکثر عمق ایمن حفاری عمودی در این ناحیه محدود به ۲–۳ متر است و برای حفاری‌های عمیق‌تر نیاز به زاویه شیب حدود ۶۰–۷۰ درجه برای تأمین ایمنی وجود دارد. نتایج اهمیت لایه‌های دانه‌درشت بین لایه‌های عمدتاً دانه‌ریز را برجسته می‌کنند، زیرا این لایه‌ها می‌توانند به‌عنوان سطح گسیختگی ترجیحی تحت شرایط بلندمدت و کوتاه‌مدت عمل کنند. تحلیل‌های بازگشتی بر روی ترانشه‌های موجود الگوهای پیش‌بینی‌شده را تأیید کردند، به‌گونه‌ای که شکست‌های مشاهده‌شده یا تغییر شکل‌های جزئی با FS مدل‌شده و توصیه‌های ناحیه‌ای همخوانی داشتند. مقایسه با مطالعات پیشین بر روی اعضای دانه‌درشت آبرفت‌های تهران نشان می‌دهد که وارد کردن پارامترهای مهندسی مانند مقاومت برشی بدون زهکشی، وزن واحد و زاویه اصطکاک به‌طور قابل توجهی دقت پیش‌بینی را افزایش می‌دهد. همچنین، مطالعه اثربخشی بهینه‌سازی مسیر لغزش بحرانی مبتنی بر روش مونت‌کارلو را نشان می‌دهد، زیرا تغییرات جزئی در سطح شروع لغزش بر الگوی نهایی شکست تأثیرگذار بود. ادغام داده‌های آزمایشگاهی، میدانی و مدل‌سازی عددی چندین بینش عملی فراهم می‌کند. اول، برنامه‌ریزی حفاری باید شامل زاویه شیب و عمق‌های حداکثری مبتنی بر ناحیه باشد تا ایمنی تضمین شود. دوم، رسوبات دانه‌ریز با عدسی‌های ماسه‌ای پراکنده نیاز به پایش دقیق و احتمالاً تقویت یا

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حفاری مرحله‌ای دارند تا ناپایداری کاهش یابد. سوم، نبود آب زیرزمینی در ترانشه‌ها، بررسی پایداری کوتاه‌مدت را ساده می‌کند، اما تحلیل‌های محافظه‌کارانه با فرض اشباع کامل برای ارزیابی ریسک، به‌ویژه در شرایط بارش یا نفوذ آب، ارزشمند باقی می‌مانند. به‌طور کلی، مطالعه نشان می‌دهد که پروفایل‌بندی ژئوتکنیکی دقیق همراه با تحلیل عددی جامع، امکان پیش‌بینی قابل اعتماد رفتار حفاری را فراهم می‌کند. این تحقیق اهمیت ادغام ویژگی‌های ناهمگن خاک در مدل‌های پایداری شیب را تأکید می‌کند و راهنمایی‌های عملی برای مهندسان عمران فراهم می‌آورد تا حفاری‌های ایمن و کارآمد در محیط‌های شهری با رسوبات آبرفتی طراحی کنند.

## نتیجه‌گیری

این مطالعه نشان می‌دهد که تکمیل طبقه‌بندی‌های زمین‌شناسی با پارامترهای مهندسی، چارچوبی مستحکم برای طراحی حفاری در رسوبات دانه‌ریز آبرفتی فراهم می‌آورد. این روش امکان شناسایی زاویه‌های بحرانی شیب، حداکثر عمق‌های حفاری و شرایط پایداری ویژه هر ناحیه را فراهم می‌کند. سه ناحیه در عضو D آبرفت‌های تهران مشخص شدند که هر یک رفتار حفاری منحصربه‌فردی دارند. تحلیل‌های بازگشتی و اعتبارسنجی میدانی، قابلیت اعتماد الگوهای پایداری پیشنهادی را تأیید می‌کنند. به‌طور کلی، نتایج ارائه‌شده راهنمایی‌های عملی برای طراحی ایمن حفاری و برنامه‌ریزی ساخت‌وساز در آبرفت‌های تهران ارائه می‌دهد و می‌تواند در محیط‌های رسوبی دانه‌ریز مشابه نیز کاربرد داشته باشد. با ترکیب آزمایش‌های آزمایشگاهی، اندازه‌گیری‌های میدانی و مدل‌سازی عددی، رویکرد مطرح‌شده در این مطالعه از کاهش ریسک، افزایش دقت پیش‌بینی و تسهیل تصمیم‌گیری برای پروژه‌های عمران شهری پشتیبانی می‌کند.

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